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INCLUDING PRESTRESSED CONCRETE

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CONCRETE AND CONSTRUCTIONAL ENGINEERING
INCLUDING PRESTRESSED CONCRETE

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EDITORIAL NOTES

Why Engineers Emigrate.

RECENT correspondence in *The Times* on the emigration of engineers to the U.S.A. was notable for the views expressed that restrictions should be placed on their freedom to do so and for suggestions that the acceptance by British engineers of appointments abroad was a new phenomenon. Throughout history men have sought better conditions in newer lands, and these emigrants have always included a large proportion of the most enterprising and ambitious of the peoples of the older countries, whether professional men seeking greater opportunities of applying their skills or manual workers seeking more regular and more remunerative employment. There is little doubt that the qualities of these immigrants and their descendants are the main reason for the higher standard of living that they have achieved.

It is against this background that we must consider the emigration to the U.S.A. of engineers from this country. They go there because the standard of living is better and there are more chances of prosperity for keen and energetic men, as others have found in the past. It is not only recently that professional men have gone abroad to earn their living. In engineering particularly this has been going on for many years. In the practice of reinforced concrete the most notable example is that of Denmark, where in the early years of this century men of outstanding ability were trained in reinforced concrete and left their own land to practise in many other countries. The development of reinforced concrete in Great Britain owes a great deal to Danish engineers who settled here earlier in this century. Similarly many engineers from this country went abroad, particularly to the Dominions and the Colonies, where many great public works stand as witnesses to their skill.

The engineer who started the correspondence said that he had recently emigrated from Great Britain and was one of forty-five engineers, about thirty years of age, from this country working for the same firm in the U.S.A. Most of them said they had emigrated because of the higher salaries in the U.S.A., where, he said, engineers were paid three times as much as in this country. This, however, is not exceptional in a country where higher incomes are enjoyed by all. For example, bricklayers and other skilled men in the building trades in New York are paid 30*s*. an hour (basic) compared with 4*s*. 8*d*. in London, which

is about seven times as much. It follows that, while professional engineers and other workers are all paid more in the U.S.A., the reward of a professional engineer in the U.S.A. would have to be seven times that of a bricklayer (say, £20,000 a year) in order to maintain his financial "status" in relation to craftsmen.

The correspondent asked why Great Britain allows engineers to emigrate. The simple answer is, of course, that we have no "iron curtain" and do not wish to have one. No figures are available of the number of engineers who have left this country in the past ten years or so, or of the number of engineers who have come here from the Continent, and even if such figures were available there would be no means of knowing whether Great Britain has gained or lost by the exchange. Also, many engineers who came to this country as refugees have since passed on to the U.S.A. and elsewhere abroad; it would be hardly true to describe these men as emigrants from a country which they used only as a staging post on their journey to the U.S.A. One would also like to know how many British engineers who have gone to the U.S.A. are men of ideas and enterprise, and how many have gone in response to advertisements asking for stress computers and detailers, and whether British engineers have introduced into the U.S.A. as many new ideas as Continental engineers have introduced here. One of the correspondents pointed out that some of these men had received their training at the expense of the British taxpayer, and that it was somewhat disloyal to take abroad the skills so acquired. This view also ignores the freedom of the individual. Also it takes no account of the skills brought to this country by engineers who were trained in the country of their birth, and again there is no way of comparing the value of the skills that come into this country with those that are taken out. Moreover no account seems to be taken of the value of the experience gained by the many engineers who return to this country.

Another correspondent suggested that able young men emigrate in order to avoid the high rates of taxation that are inseparable from a welfare state as we know it in the United Kingdom: "They realise that however hard they work, and however high they may rise, their standard of living will never be very much more comfortable than that of the inefficient and the plain lazy who can count upon security from the cradle to the grave." This is another way of saying that enterprising people emigrate, as they have always done, to a country where there are better prospects, whether the poorer prospects at home are due to unemployment, to low rates of pay, or to high rates of tax collected from those with higher incomes in order to subsidise those with smaller incomes or to maintain an inflated bureaucracy. It was claimed by another correspondent that if we are "to maintain our superiority in engineering design and execution our most highly-qualified young engineers must be given every encouragement to remain here or to return after a spell of experience abroad". This seems to make an assumption that would be difficult of proof, for the many books published recently on civil and structural engineering works in the U.S.A., Italy, Spain, Germany, and elsewhere do not suggest that at present this country has any superiority over some other countries in imaginative design and construction—rather the contrary. For this reason it could better be claimed that all our young engineers should go abroad as a matter of course on a kind of travelling scholarship in order to broaden their minds and gain experience of new developments abroad.

Structures with Self-supporting Reinforcement.

By PROFESSOR M. MENCL

TECHNICAL UNIVERSITY, PRAGUE.

MUCH attention is being paid in Czechoslovakia to methods of reducing the quantities of timber shuttering required for reinforced concrete structures, and of applying industrial methods to the process of erection. One well-proved method is the use of a self-supporting framework of reinforcement from which wooden shuttering is suspended. This method, suggested by Professor Melan, has been used throughout the world for large reinforced concrete arch bridges, especially during the years following the first world war. However, it eliminates only the temporary supporting structure, without reducing the quantity of timber required for the shuttering.

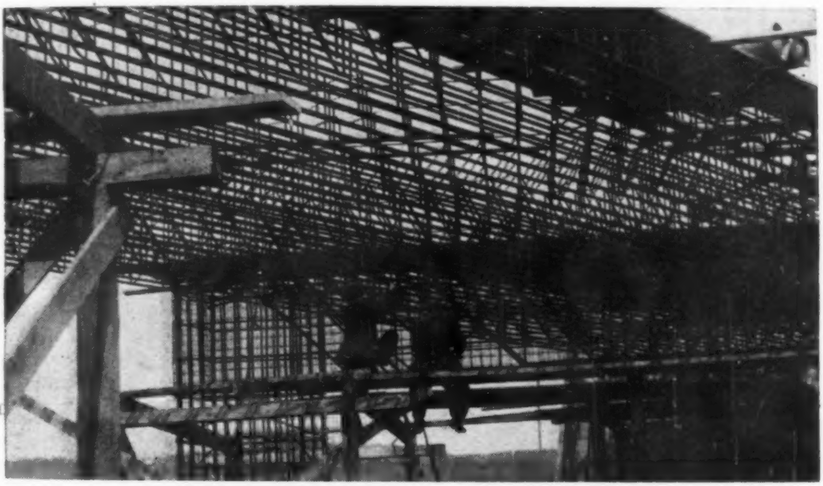


Fig. 1.—Reinforcement for a Bridge.

From 1953 onwards tests were made in Czechoslovakia on the erection of monolithic reinforced concrete structures with welded self-supporting frames of reinforcement to which steel netting with a mesh of about $\frac{1}{8}$ in. (4 mm.) square is attached. The wires of the netting are at $\frac{1}{5}$ in. (5 mm.) centres, and have a diameter of about 1 mm.; the weight is about 0·41 lb. per square foot. From 1954 onwards this method was used for the roofs of several houses, for two bridges (one of which is shown in *Fig.* 1), and for two reinforced concrete caissons for the foundations of a railway bridge. In 1957 its use was extended to other structures.

The system offers many advantages in addition to avoiding the use of almost all the shuttering and temporary supports. The welded frameworks may be prefabricated so that the work on the site consists only of erection and concreting. The structure has all the advantages of monolithic construction. The frames are comparatively light and can be erected without expensive equipment.

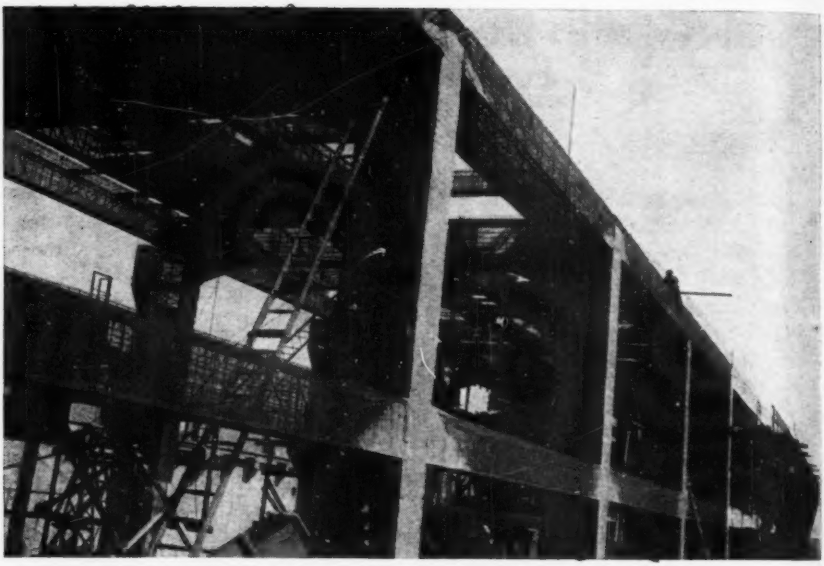


Fig. 2.—Reinforcement for Frame of a Turbine House.

Fig. 3.—Reinforcement for Frame of a Turbine House.

Concrete placed in the netting has been found to be of better quality than concrete placed in timber shutters.

For example, a load test on the over-bridge shown in *Fig.* 1 produced a deflection of 1/11,200 of the span, although a substantially greater deflection was expected. The increased strength of the concrete is due to the fact that, after a comparatively negligible quantity of mortar has fallen through the meshes of the netting, the surplus water filters through the compacted concrete and drips, perfectly clear, through the netting; the consequent reduction in the water-cement ratio increases the strength of the concrete. The extent of this improvement in strength is now being investigated, but this is not the only reason for the good results obtained in the load tests.

The co-operation of the concrete and reinforcement is not achieved by bond

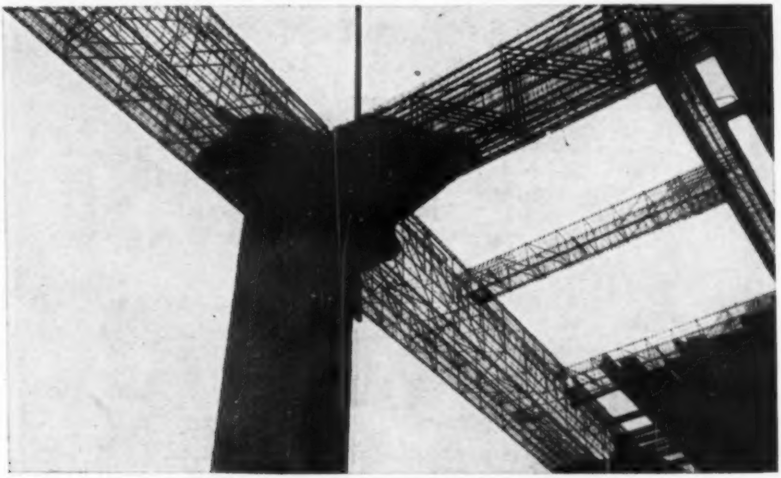


Fig. 4.—Junction of Column and Beams.

alone; the abutting against the concrete of the diagonals and vertical members of the vertical and horizontal lattice girders at the level of the top and bottom chords of the structure is at least as important. This, together with the high load-bearing capacity of the girders, is the reason why the structure is more elastic than normal reinforced concrete structures; the deflections are entirely elastic and not permanent. The co-operation between the concrete and the reinforcement is ensured in a manner similar to that of the Austrian reinforcement comprising two bars of high-grade steel linked together (" Bi-Stahl "), and as it is not dependent on bond alone the reinforcement may consist of a small number of large steel bars. Members reinforced in this way can be concreted much more easily, and the cost of welding the reinforcement is less.

In many countries rolled steel sections are often used as self-supporting reinforcement, but in Czechoslovakia steel bars are used for all structures designed by the writer and erected by this method. In particular, " Roxor " steel [a low-alloy steel with good welding properties, of cruciform section, and with a

minimum yield stress of about 54,000 lb. per square inch (3800 kg. per square centimetre)] in diameters of up to about 3 in. is used.

Three structures completed in the year 1957 are described in the following.

A Turbine House at a Power Station.

Figs. 2, 3, and 4 show the framework for a turbine house at a power station. The structure was designed for erection by conventional methods, and the only modifications made were the use of self-supporting reinforcement and of precast slabs instead of a roof cast in place. The longitudinal reinforcement of the columns was little changed; in place of stirrups, however, a lattice of about $\frac{5}{8}$-in. bars was used. This was enclosed by welded fabric reinforcement, com-

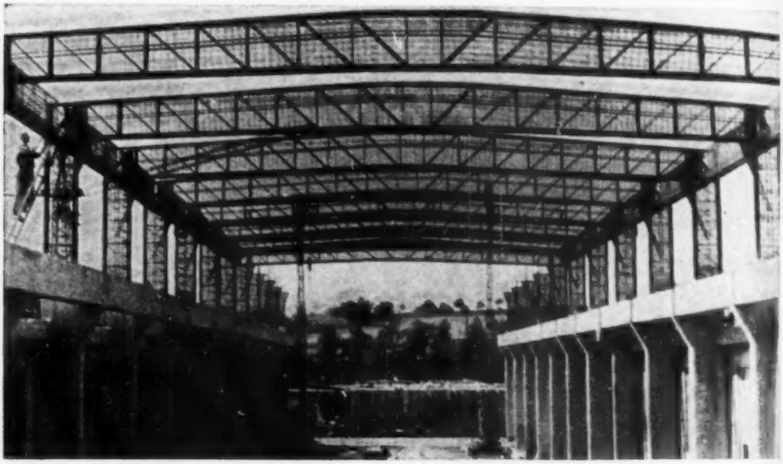


Fig. 5.—Construction of a Curing Chamber.

prising bars of about $\frac{1}{8}$ in. diameter at about 4 in. centres to which the netting was fixed. (The impression given by *Figs.* 2 and 4 that the bars in the columns are still visible after the concrete is placed is an illusion created by variations in the density of the concrete and the angle of incidence of the light.) The steel fabric is used to obtain the necessary cover for the longitudinal reinforcement, and to provide a sufficient number of places for the attachment of the netting, which must be prevented from bulging. When using this particular fabric, every square yard of the netting is fastened by about 80 ties of binding wire; experience shows that this is sufficient.

The reinforcement of the main beam consists of five lattice girders with upper and lower chords of $1\frac{1}{2}$-in. Roxor steel. Larger diameters were not available at the time, otherwise two girders with about 2-in. chords could have been used. The separate frameworks were welded at a nearby workshop, and the greatest weight of a column-frame was about 4850 lb. The frames for the columns were erected by a small crane and those for the beams by hoisting tackle on supporting trestles. After the column-frames had been temporarily set in place, the longitudinal reinforcement was welded to bars protruding from the floor.

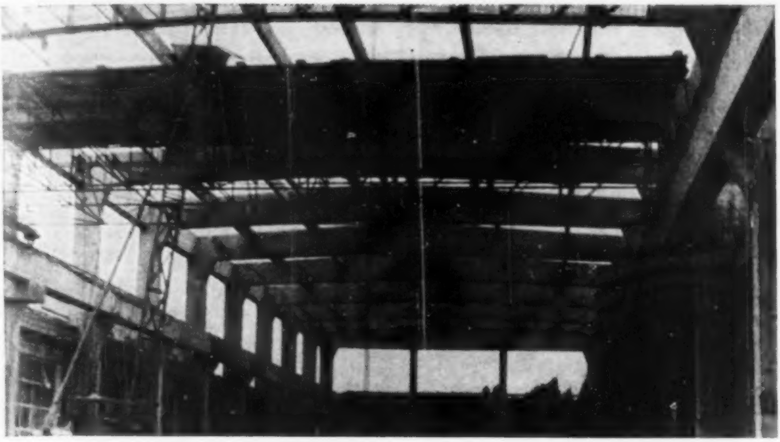


Fig. 6.—Concreting Beams.

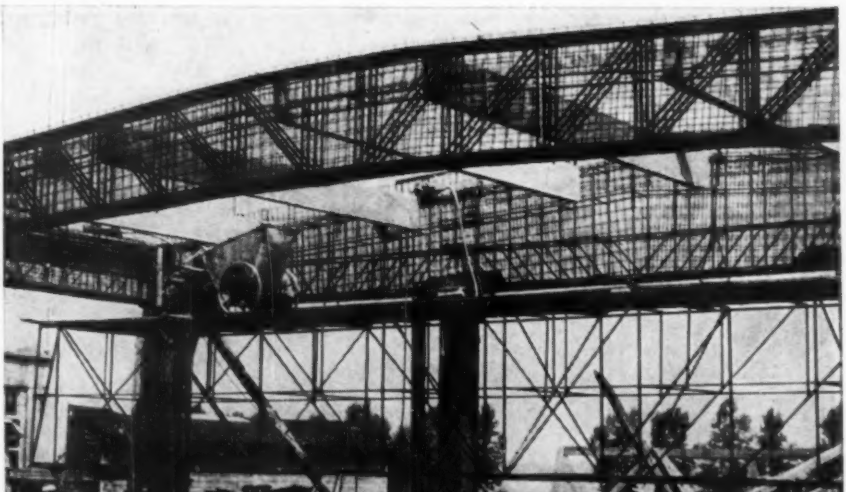


Fig. 7.—Arrangement for Placing Concrete in Beams.

After all the framework had been erected the reinforcement was welded at all intersections, and the main bars of the columns were welded to the reinforcement of the beams. It was not assumed, however, that this in itself would constitute a rigid joint, or that the reinforcement of the columns would be able to support the weight of the concreted beam. The columns and joints were concreted first (*Figs.* 2 and 4), and the beams were concreted about 48 hours later, when the joints had hardened.

This method as well as the usual method of construction were used in this structure, and the use of self-supporting reinforcement was appreciably cheaper.

Curing Chamber for Precast Concrete.

A factory for the production of precast panels for concrete houses was extended by erecting a single-story building of about 40 ft. span between two existing workshops. As no carpenters were available and timber for shuttering was not readily obtainable it was decided to use self-supporting reinforcement, and this decision proved to be justified. Welding equipment was readily available, and the structure was erected within two months by a few men; there was little interference with the production of the factory.

The structure is shown in *Figs.* 5, 6, and 7. The frames were spaced at the same centres as those of the existing workshops. Hinges were formed at the bases of the columns and the columns are supported on short beams which transmit the load to bases built on both sides of the existing bases. As the atmosphere of the curing chamber will be humid and warm, insulated roofing supported on precast purlins at about 6 ft. 6 in. centres is used. The self-supporting reinforcement is of Roxor steel, and is generally similar to that described previously.

As only small cranes were available the method of erection was as follows. The frameworks of all the columns were erected and connected with the frameworks of the crane beams. The columns were secured by stays, and were concreted, together with the crane beams, before the frameworks for the roof beams were erected. A travelling crane with a carrying capacity of about 1 ton,

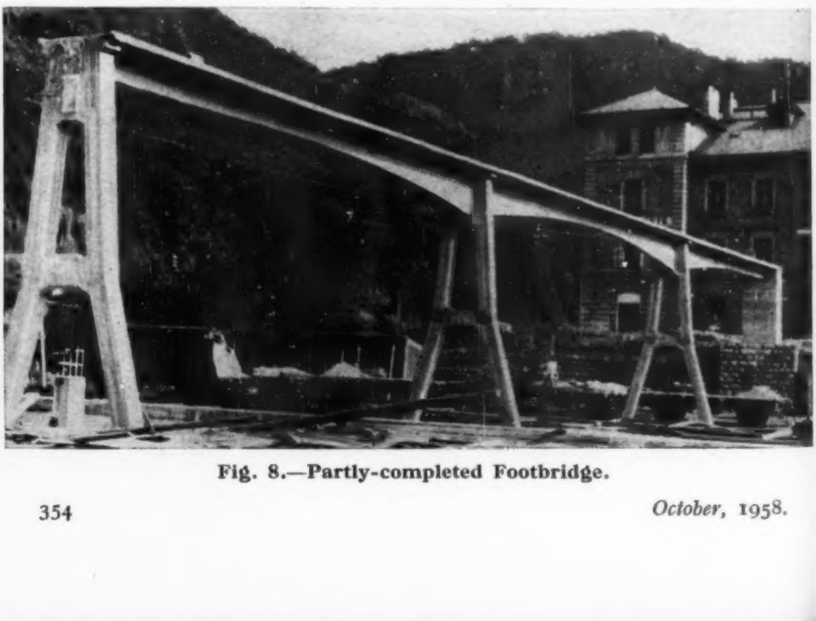


Fig. 8.—Partly-completed Footbridge.

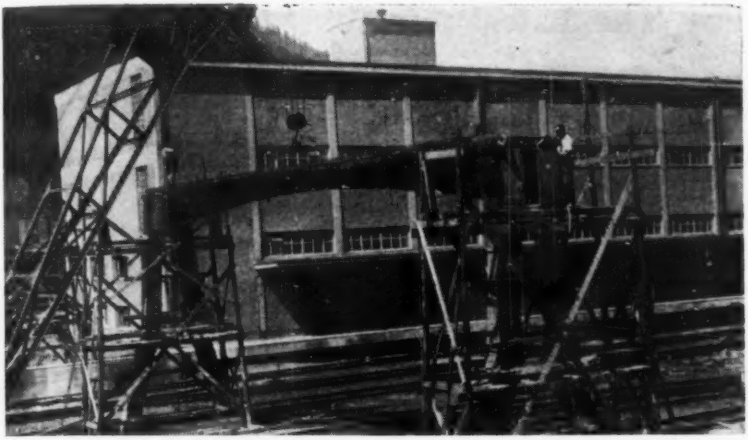


Fig. 9.—Erecting Prefabricated Reinforcement.

fabricated from reinforcing steel, was erected on the crane beams which were covered with a service platform. The purlins were then placed in position and the concreting was completed from the service platform.

A Footbridge.

A footbridge at a railway station (*Figs.* 8 and 9) is formed by a continuous T-shaped beam with three non-prismatic spans; the end spans are each about 51 ft. and the middle span about 60 ft. As it was possible without difficulty to support the beam during the concreting operations by means of temporary struts between the rail-tracks, a part only of the reinforcement was welded to form two self-supporting lattice girders, both chords being of Roxor steel. The rest of the reinforcement was inserted separately. The reinforcement for each span was fabricated separately, enclosed in steel mesh, covered with netting, and supplemented by other reinforcement. The three beams thus formed were transported to the site, where a railway crane hoisted them into place on previously-prepared supports (*Fig.* 9). After inserting the additional reinforcement required to resist the support moments and welding together the bars of adjoining chords of the girders, concreting was carried out without difficulty. A railway crane was used to hoist the concrete in barrows.

The experience with this bridge, and with the work shown in *Fig.* 1, proved the advantage of this method of construction for railway works. Interference with the rail-traffic is greatly reduced, and the time required for erection is short. The falsework may be removed shortly after the concrete is placed, as its principal purpose is the reduction during concreting of the compressive stresses in the bars of the framework, which might otherwise buckle. The time needed for concrete to harden before the removal of shuttering and supports is saved. Plastering and other finishing work may be done immediately after the concreting is completed, and the structure can be used much earlier than a structure built by ordinary methods.

The Late Professor Kleinlogel.

PROFESSOR A. KLEINLOGEL, who died on June 17, 1958, at Darmstadt, Germany, was one of the last of the engineers who did pioneer work in the development of reinforced concrete in the early years of the century. He was born in 1877, graduated in 1900 at the Technical University at Stuttgart, and worked for many years with Bach and Mörsch, two other pioneers of concrete. His first paper to attract general attention was published in 1903; it dealt with the extensibility of concrete and reinforced concrete, and in it he refuted Considère's idea that the extensibility of reinforced concrete was many times greater than that of plain concrete. In 1910 he was awarded a degree of Doctor of Engineering at the Technical University of Dresden for a thesis on the bond between steel and concrete, and in 1912 was appointed a lecturer at the Technical University of Darmstadt, where he also practised as a consulting engineer. His approach to reinforced concrete was a very practical one and his many books and other publications dealt with a very wide range of theoretical and practical problems.

Kleinlogel is best known for his "Rahmenformeln" (Formulæ for Rigid Frames), the first edition of which was published in 1914. This book is now in its twelfth edition, and has been translated into English, French, and Italian; in nearly every later edition new formulæ have been added, and the present edition bears very little resemblance to the first. "Rahmenformeln" was followed after the first world war by "Mehrstielige Rahmen" (Continuous Frames), "Durchlaufträger" (Continuous Beams), and "Belastungsglieder" (Loading Terms) now in its eighth edition. Kleinlogel's other books include works on expansion joints, concreting in cold weather, prefabrication, and the effects of chemical, mechanical, and other influences on concrete. In 1922 he followed the late Dr. Emperger as editor of "Beton u. Eisen", and under his editorship the influence of the journal increased and it also became widely read abroad; he resigned from this post in 1943.

Kleinlogel was one of the early protagonists of the strict control of work on the site and of training courses for site engineers and building operatives (similar to those now arranged in this country by the Cement & Concrete Association). He acted as an arbitrator and as a technical expert in many court cases. During the years 1929 to 1931 he designed several factories in the U.S.S.R., and spent some time as technical adviser in that country.

In 1951 he was honoured by the Emil Mörsch medal, the highest distinction in Germany for achievements in reinforced concrete.

As a university teacher he was very popular because of the clarity of his lectures and the personal interest he took in his students. His sense of humour and pleasant manners made his companionship very enjoyable. He was a keen sportsman, fond of ski-ing and tennis. He led an active life until a few years before his death, and when he was more than fifty years of age he was awarded a gold medal for all-round efficiency in sports; although a very hard worker he always found time for recreation. His passing leaves the writer and many other friends and collaborators with a very real sense of loss.

K. HAJNAL-KÓNYI.

A Gasholder Tank in London.

A TANK and foundation for a gasholder with a capacity of about 5,000,000 cu. ft. is now being built at the Wandsworth works of the South Eastern Gas Board. The arrangement of the tank is shown in *Fig.* 2, and details of the grillage which will support the crown of the gasholder are shown in *Figs.* 3 and 4.

The diameter of the tank is about 220 ft., and its greatest depth is 37 ft. The ground consists of a layer of filling 8 ft. deep, which overlies layers of sandy clay and stones 4 ft. thick. Below these are a layer of ballast 15 ft. thick and firm blue clay at a depth of 27 ft. The walls of the tank consist of two concentric rings of steel sheet piling ; the outer ring was driven before excavation was commenced, and formed a retaining wall within which the ground was excavated to a depth of about 18 ft. The inner ring was then driven from this level ; the stages of construction are shown in *Fig.* 2.

When completed, the tank will be filled with water to seal the gasholder ; the joints between the sheet piles were therefore caulked with cement and asbestos cord to render them watertight. Upper and lower capping beams are provided ; the lower beam (*Fig.* 1) is of reinforced concrete and the upper beam is prestressed by means of 80 alloy-steel bars of $1\frac{1}{8}$-in. diameter so as to permit the ground outside the tank to be removed if necessary without emptying the tank.

The loads from the gasholder are supported by the inner ring of steel sheet piles, the outer ring acting only as a retaining wall. The loads are applied through roller-carriages fixed to the upper ring-beam and slab, which are anchored by means of prestressing cables to ninety-six supporting piers spaced uniformly around the perimeter of the tank. The piers are supported on the lower ring-beam. Inlet and outlet pits about 48 ft. deep are provided.

Fig. 1.—Casting Lower Ring-beam.

Within the tank the ground is excavated to a slope of 60 deg. The precast members which will support the crown of the gasholder rest on ninety-two prestressed precast piles 16 in. square and 45 ft. long. Circular pile-caps of the type shown in *Fig.* 4 were cast in place, and precast cantilevered beams were bolted to the caps to support segmental precast purlins.

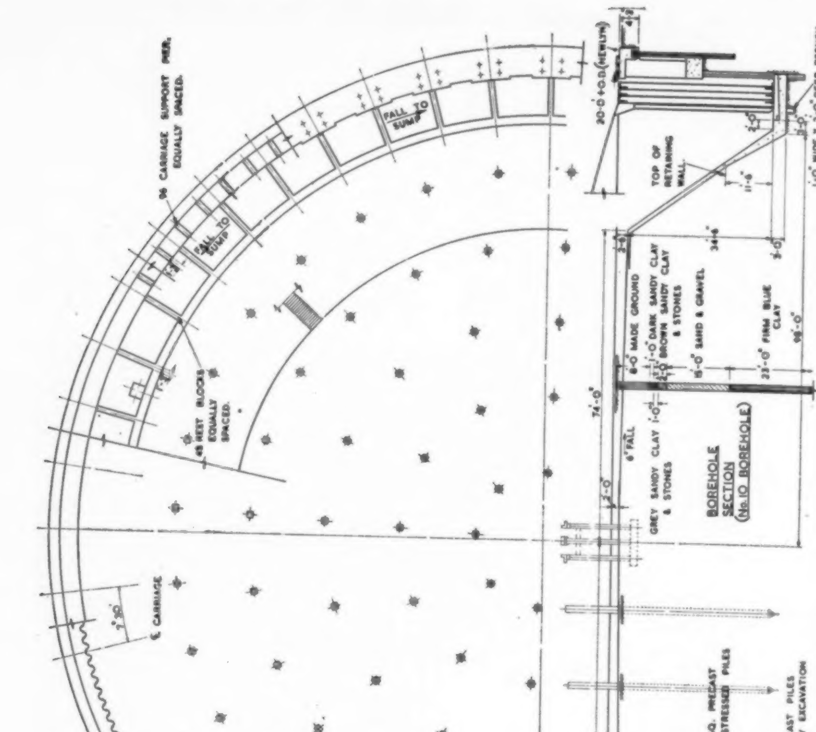


Fig. 2.—General Arrangement of Foundations.

The purlins were wedged in position by means of timber packing-pieces, which also acted as shuttering for the joints which connect the purlins and which were cast in place. Every third ring of purlins is prestressed in order to ensure the stability of the structure, and the cantilevers are also connected along two normal diameters of the tank.

The work was started in November 1957 and is expected to be completed by November 1958. The sequence of construction was as follows.

(1) The outer ring of steel sheet piling was driven with the aid of a mobile crane (*Fig.* 5).

(2) The capping beam for the piles was cast while the prestressed piles were being driven. A mobile crane (*Fig.* 5) was used to drive the prestressed piles; the weight of the falling hammer was 4 tons and the piles were driven to a set of ⅜ in., the fall of the hammer being 3 ft. 6 in.

(3) The ground inside the tank was excavated to the level of the lower ring-beam. A temporary bridge (*Fig.* 6) was erected to provide access to the interior of the tank.

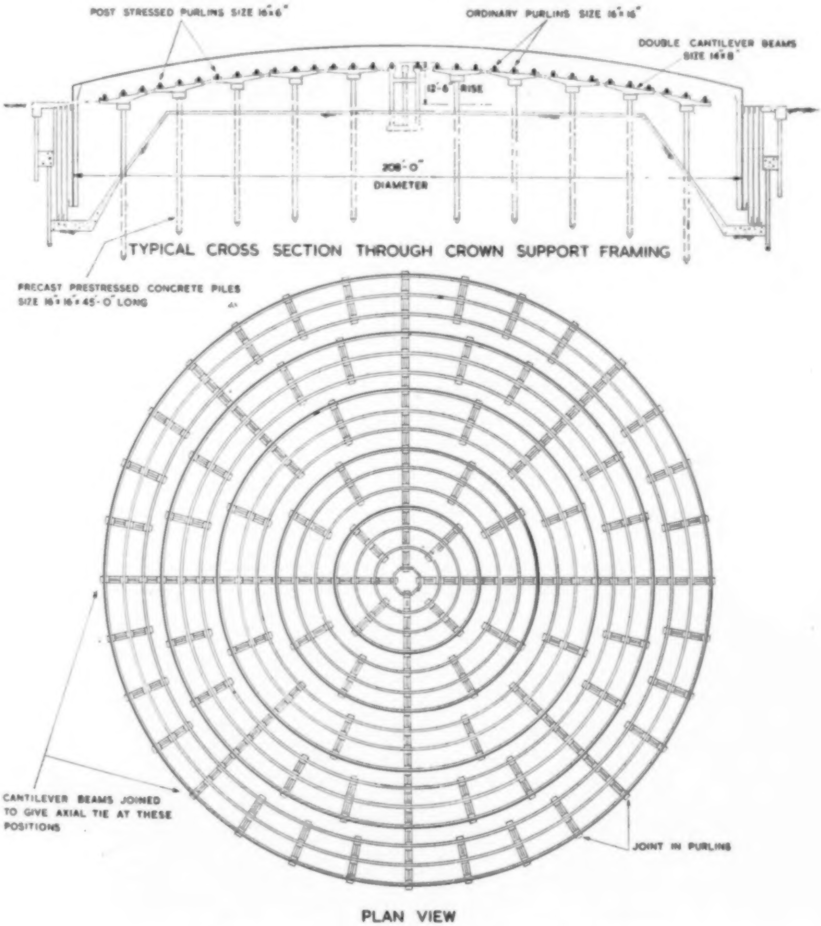
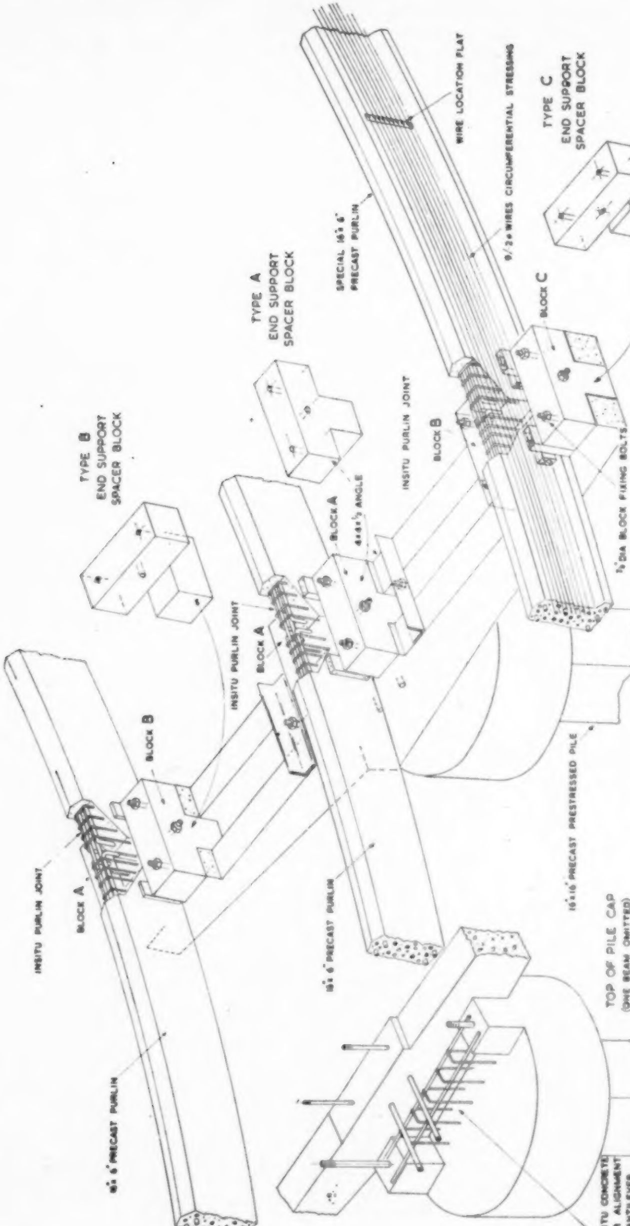


Fig. 3.—Arrangement of Supports for Crown.



CROWN SUPPORT FRAMING — CANTILEVER ASSEMBLY

Fig. 4.—Details of Supports for Crown.

(4) The inner ring of steel sheet piling was driven.

(5) The supporting piers were cast and prestressed.

(6) The excavation was completed and the base of the tank concreted ; the slopes were also lined with concrete 6 in. thick (*Fig.* 7).

(7) The grillage to support the crown of the gasholder was built.

All the steel sheet piles with the exception of those used to form the inlet and outlet chambers were driven by means of a Diesel hammer. The sheet piles forming the inlet and outlet chambers are 60 ft. long. They were driven by means of a steam hammer, the steam for which was produced by a new type of steam generator capable of providing the full working pressure five minutes after being filled with cold water. The excavation was kept dry by means of a portable and submersible electrically-driven pump of Swedish design, which is claimed to be smaller and lighter than the pumps of equivalent capacity now in general use in this country.

The arrangement of the concrete-mixing plant is shown in *Figs.* 8 and 9. Four stock-piles of aggregates are provided corresponding to the four compartments of the weigh-batcher. Two of the compartments are used for $\frac{3}{4}$-in. to $\frac{3}{16}$-in. river gravel, one for washed sand, and one for ballast. The aggregate is delivered by road, and the stock-piles are trimmed by means of an electrically-operated hand-scraper. Sulphate-resisting cement, which is used throughout, is delivered by road and stored in a silo. The water-cement ratio is 0·5 and the proportions of the concrete are 1 : 2·2 : 4·2 by weight. The aggregates pass through manually-operated doors to the

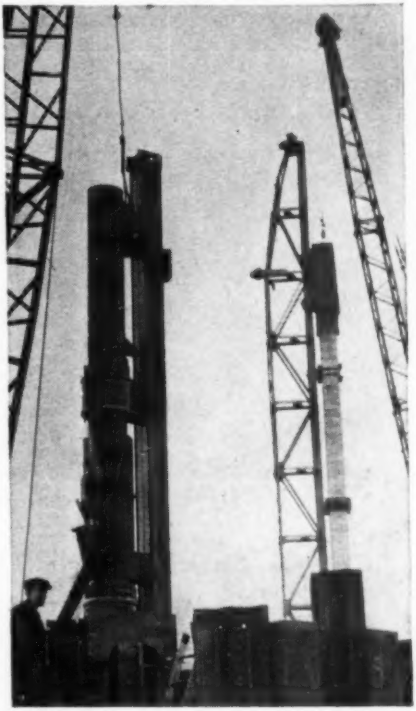


Fig. 5.—Driving Steel and Prestressed Concrete Piles.

Fig. 6.—Temporary Bailey Bridge.

Fig. 7.—Placing Concrete on a Slope of 60 deg.

Fig. 8.—Arrangement of Mixing Plant.

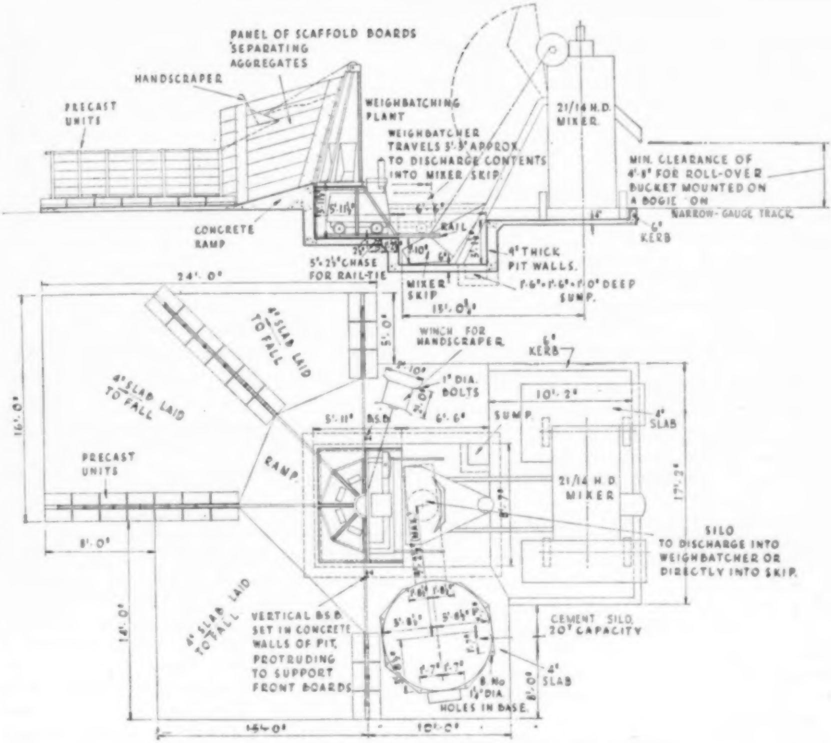


Fig. 9.—Arrangement of Concrete Mixing Plant.

hopper of the weigh-batcher ; the hopper is then pulled over the mixer-skip by means of an electric motor and the contents automatically discharged. A 21/14 mixer is used, and the concrete is discharged into crane skips which are carried on flat bogies. The bogies are mounted on a narrow-gauge track which surrounds the tank, and are hauled by a Diesel-engined locomotive. The skips are lifted from the bogies and placed where required by means of a mobile crane.

The specified compressive strength of the concrete at 28 days is 4000 lb. per square inch, and the average cube strength is about 5500 lb. per square inch. At the commencement of the work one set of cubes was made every day, but the number was later reduced to two sets per week. Each set comprises seven cubes, of which three are tested by the contractor and four by the South Eastern Gas Board. Compacting-factor and sieve tests are made regularly.

The cost of the work is about £118,000. The design was prepared by the South Eastern Gas Board, and the contractors for the civil engineering work are the Demolition & Construction Co., Ltd. The licensees for Bailey bridges are Messrs. Thos. Storey (Engineers), Ltd.

Book Reviews.

"Design of Eccentrically-loaded Columns by the Load-factor Method." By J. D. Bennett. (London: Concrete Publications, Ltd. Price 8*s.*; 8*s.* 6*d.* by post. $1.90 in Canada and U.S.A.)

THIS is a reprint of four articles which appeared in this journal in 1957 and 1958. The method described is a refinement of that given in British Standard Code of Practice No. 114 of 1957, and twenty charts are included by means of which any column subjected to direct load and bending about one axis of symmetry may be rapidly designed.

Examples show the application of the method to columns reinforced with mild steel or cold-worked bars and with symmetrical or unsymmetrical reinforcement. The simplicity of the method makes it possible to prepare several alternative designs in a few minutes.

B.S. 1000 (69): 1958. Universal Decimal Classification: Building. (London: British Standards Institution. Price 15*s.*)

THE fourth international edition, in English, of the Universal Decimal Classification is being prepared by the British Standards Institution, and is authorised by the Fédération Internationale de Documentation. B.S. 1000 (69) is concerned with building, as distinct from civil engineering and architecture. It provides a system of classification for documents relating to building materials, components, and methods.

"Design of Concrete Structures." By L. C. Uruquhart, C. E. O'Rourke, and G. Winter. (London: McGraw-Hill Publishing Co., Ltd. 1958. Price 62*s.*)

BASICALLY the same as the previous edition reviewed in this journal in January, 1955, the present edition of this deservedly well-known U.S.A. book conforms with the current regulations of the American Concrete Institute. The most important alteration is the addition of a chapter on the ultimate-load method of design treated from the point of view of the U.S.A. regulations, and including a discussion of the principles on which the method is based. An extensive set of graphs, taken from an article by Messrs. Whitney and Cohen in the A.C.I. Journal, facilitates application of the ultimate-load method to eccentrically-loaded rectangular and circular columns; it will be noticed that the minor corrections to the diagrams on the graphs relating to circular columns, published in the A.C.I. Journal last year, have not been made in the book. A good feature of these graphs, and indeed of others in U.S.A. books, is the inclusion of curves for square columns with the longitudinal bars arranged in a circle. This allows an appreciable reduction in the number of hoops required in columns where the bending moment is small compared with the direct load.

"Kugelschlagprüfung von Beton mit dichtem Gefüge Einfluss des Prüfalters." By Kurt Gaede. (Berlin: Wilhelm Ernst & Son. Price 6 D.M.)

IN this booklet, published on behalf of the German Committee on Reinforced Concrete, the author describes tests to determine the influence of the age of concrete upon the ball-impact test for assessing compressive strength. Since the rate of hardening of the surface of concrete in structures is faster than the rate of increase of the strength of test cubes, data derived from tests on cubes at 28 days need to be adjusted when applied to tests at a later date in accordance with the following percentages of the strength of cubes at 28 days; 90 days, 94 per cent.; 365 days, 75 per cent.; 730 days, 64 per cent. The results of the tests are summarised.

"Grundlagen des Stahlbaues." By Fritz Stüssi. (Berlin: Springer Verlag. Price D.M. 55.50.)

THIS is a treatise on the principles of construction in steel: special attention is given to the design of thin steel walls. The author is one of the foremost authorities on steel construction, and it is interesting to note that he expresses the view that modern long-span box-girder bridges have the same constructional characteristics as those adopted by Robert Stephenson for the Britannia Bridge. The first illustration in the book shows the cast iron bridge over the river Severn at Coalbrookdale built by Abraham Darby.

Gable Frames with Multiple Bays.

By V. A. MORGAN, M.Eng., A.M.I.C.E.

By the use of tables similar to *Table* I, gable frames with multiple bays can be analysed rapidly with the aid of simple arithmetic and a slide-rule. As will be shown, equations for the bending moments, reactions, and deflections can be obtained which consist of a factor multiplied by a fraction of which the numerator and denominator are groups of stiffness-ratios K multiplied by the ratio ϕ of the rise of the ridge to the height of the column raised to some power. These equations can be arranged in tabular form for particular types of load, and numerical solutions obtained by substituting suitable values for the frame considered.

A Gable Frame with Four Bays.

The frame shown in *Fig.* 1 supports a uniformly-distributed load of w lb. per foot. The column-bases are fixed, and all the joints are rigid except those between the rafters and the tops of the internal columns, which are hinged. The stiffness-ratios of the members are expressed in terms of the stiffness of the rafter $\frac{I_o}{s_o}$; for the rafters $K_o = \frac{I_o}{s_o} \div \frac{I_o}{s_o} = 1$, and for the columns

$$K_1 = \frac{I_1}{h} \div \frac{I_o}{s_o},\ K_2 = \frac{I_2}{h} \div \frac{I_o}{s_o},\ \text{and}\ K_3 = \frac{I_3}{h} \div \frac{I_o}{s_o},$$

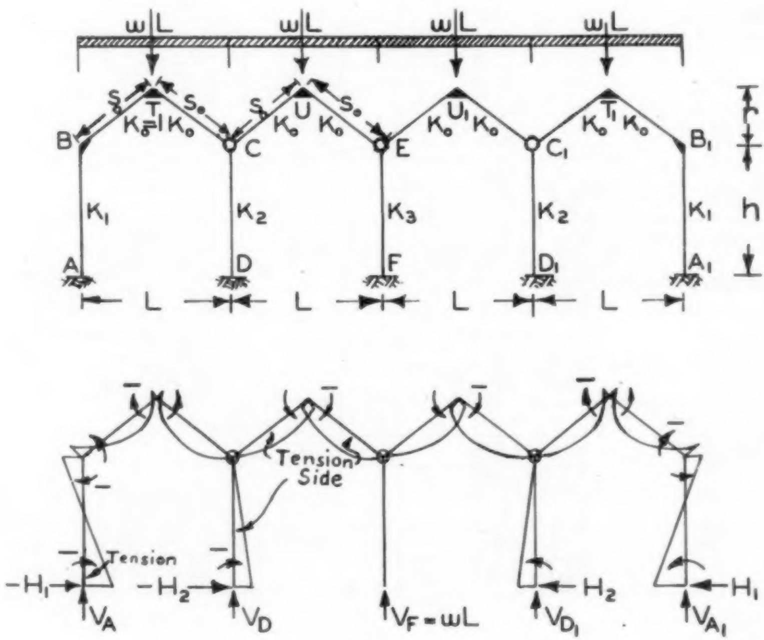


Fig. 1.

TABLE I.

$F = \frac{wL^2}{8d_5}$		1	K_1	$\frac{1}{K_1}$			1	K_1	$\frac{1}{K_1}$
$-M_{AB}$	1	4			$+2K_2\phi^2$	1	4		
	ϕ	75	88			ϕ	15	8	
	ϕ^2		43			ϕ^2		−1	
$-M_{BA}$	1	8			$+2K_2\phi^2$	1	8		
	ϕ	75				ϕ	15		
	ϕ^2		−43			ϕ^2		1	
$-H_1 h$	1	12			$+2K_2\phi^2$	1	12		
	ϕ	150	88			ϕ	30	8	
	ϕ^2					ϕ^2			
M_{TB}	1	4		3	$+2K_2\phi^2$	1	4		3
	ϕ	−31·5				ϕ	−1·5		
	ϕ^2	−102	−43·5			ϕ^2	−6	−1·5	
$V_A L - \frac{wL^2}{2} = -V_C L + wL^2$	1	8			$+2K_2\phi^2$	1	8		
	ϕ	75				ϕ	15		
	ϕ^2		−43			ϕ^2		1	
M_{UC}	1	−8		−3	$+2K_2\phi^2$	1	32		12
	ϕ	−6				ϕ	72		
	ϕ^2	+102	+65			ϕ^2	96	28	
$-\frac{H_2 h}{K_2\phi} = \frac{-M_{DC}}{K_2\phi}$	1	80		30	$V_D = V_C$ $V_F = wL$				
	ϕ	156			$\Delta_E = 0$				
	ϕ^2	180	54						
$H_r h$ = rafter thrust in CUE	1	12			$+2K_2\phi$	1	40		15
	ϕ	150	88			ϕ	90		
	ϕ^2					ϕ^2	120	35	
$\frac{-3EI_o\Delta_1}{h\,s_o}$	1				$+2K_2\phi^2$	1			
	ϕ	88		37·5		ϕ	8		7·5
	ϕ^2	64·5				ϕ^2	−1·5		
$\frac{-3EI_o\Delta_2}{r\,s_o}$	1	80		30	$\Delta_1 = \Delta_B$				
	ϕ	156			$\Delta_2 = \Delta_C$				
	ϕ^2	180	54						
d_5	1	8		3	$+2K_2\phi^2$	1	8		3
	ϕ	18				ϕ	18		
	ϕ^2	48	23			ϕ^2	24	7	

TABLE II.

COLUMN NOS.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
$F = \frac{wL^2}{8d_5} = \frac{M}{d_5}$		1	$K_1 = 2$	$\frac{1}{K_1} = 0{\cdot}5$	SUM × 1 × 3/7 ETC.	$\frac{K_2 I}{2\phi^2}$	1	1	$K = 2$	$\frac{1}{K_1} = 0{\cdot}5$	SUM × 1 × 3/7 × 9/49	SUM (11) × 18/49	TOTAL (5)+(12)	(13) × $\frac{M}{d_5}$
d_5	1	8		1·5	9·5		1	8		1·5	9·5			
	3/7	18			7·7	$\frac{18}{49}$ ×	3/7	18			7·7			
	9/49	48	46		17·3		9/49	24	14		7·0			
					34·5						24·2	8·9	43·4	= d_5
$-M_{AB}$	1	4			4		1	4			4			
	3/7	75	176		107·6	$\frac{18}{49}$ ×	3/7	15	16		13·3			
	9/49		86		15·8		9/49		−2		−0·4			
					127·4						16·9	6·2	133·6	3·07M
$-M_{BA}$	1	8			8		1	8			8			
	3/7	75			32·1	$\frac{18}{49}$ ×	3/7	15			6·5			
	9/49		−86		−15·8		9/49		2		0·4			
					24·3						14·9	5·5	29·8	0·685M
$-H_1 h$	1	12			12		1	12			12			
	3/7	150	176		140	$\frac{18}{49}$ ×	3/7	30	16		19·7			
					152						31·7	11·7	163·7	3·755M
M_{TB}	1	4		1·5	5·5		1	4		1·5	5·5			
	3/7	−31·5			−13·6	$\frac{18}{49}$ ×	3/7	−1·5			−0·6			
	9/49	−102	−87		−34·7		9/49	−6	−3		−1·8			
					−42·8						3·1	1·14	−41·8	−0·96M
$V_A L - \frac{wL^2}{2} = -V_C L + \frac{wL^2}{2}$	1	8			8		1	8			8			
	3/7	75			32·1	$\frac{18}{49}$ ×	3/7	15			6·5			
	9/49		−8		−15·8		9/49		2		0·4			
					24·3						14·9	5·5	29·8	0·685M
M_{UC}	1	−8		−1·5	−9·5		1	32		6	38			
	3/7	−6			−2·6	$\frac{18}{49}$ ×	3/7	72			30·8			
	9/49	102	130		42·6		9/49	96	56		29·8			
					30·5						98·6	36·2	66·7	1·535M
$H_r h$ = rafter thrust in CUE	1	12			12		1	40		7·5	47·5			
	3/7	150	176		97	⊗ $\frac{6}{7}$ ×	3/7	90			38·5	⊗ × 6/7		
	9/49						9/49	120	70		34·9			
					109						120·9	101·8	210·8	5·02M
$\frac{-3EI_0 \Delta_1}{h s_0}$	1						1							
	3/7	88		18·75	45·6	$\frac{18}{49}$ ×	3/7	8		3·75	5·0			
	9/49	64·5			11·9		9/49	−1·5			0·3			
					57·5						5·3	0·97	58·47	1·35 M
$\frac{-3EI_0 \Delta_2}{r s_0}$	1	80		15	95	$-\frac{\Delta_2}{M} = \frac{4{\cdot}96\, r s_0}{3EI_0}$, $-\frac{\Delta_1}{M} = \frac{1{\cdot}35\, h s_0}{3EI_0} = \frac{3{\cdot}16\, r s_0}{3EI_0}$								
	3/7	156			66·8									
	9/49	180	108		52·8									
					214·6								214·6	4·96 M
$-\frac{H_2 h}{K_2 \phi} = \frac{-M_{DC}}{3/7}$	1	80		15	95	$V_D = V_C$ $V_F = wL$ $\Delta_1 = \Delta_B, \Delta_2 = \Delta_C$								
	3/7	156			66·6	$\Delta_E = 0$								
	9/49	180	108		52·8									
					214·6	× 3/7 = $H_2 h = M_{DC}$ = 91·8								2·12 M

in which I_0, I_1, I_2, and I_4 are the moments of inertia of the members BT, AB, CD, and EF.

EXPLANATION OF THE TABLE.—For this type of frame and load, the bending moment at A in column AB is given by

$$-M_{AB}=\frac{wL^2}{8}\times\frac{4+75\phi+88K_1\phi+43K_1\phi^2+2K_2\phi^2(4+15\phi+8K_1\phi-K_1\phi^2)}{8+\frac{3}{K_1}+18\phi+48\phi^2+23K_1\phi^2+2K_2\phi^2\left(8+\frac{3}{K_1}+18\phi+24\phi^2+7K_1\phi^2\right)}$$

$$=\frac{wL^2}{8}=\frac{\text{numerator}}{d_5}.$$

The denominator d_5 is common to the equations for all the other moments, reactions, and deflections, and can be combined with $\frac{wL^2}{8}$, so that $-M_{AB}=F$ times the numerator, in which $F=\frac{wL^2}{8d_5}$.

The numerator for $-M_{AB}$ is shown in the first part of *Table* I arranged in columns under the heading 1, K_1, and $\frac{1}{K_1}$, and lines to be multiplied by 1, ϕ, and ϕ^2. If the section for $-M_{AB}$ is compared with the numerator of the foregoing equation the arrangement will be clear; the second part of the numerator is multiplied by the factor $2K_2\phi^2$ shown in column (6) of *Table* I. The common denominator d_5 is shown in the bottom part of *Table* I, and the remainder of this table gives the numerators for other quantities.

USE OF TABLE.—A copy of *Table* I is made with the values of ϕ, ϕ^2, K_1, and K_2 inserted, as shown in *Table* II in which $\phi=\frac{3}{7}$, $K_1=2$, and $K_2=1$, with additional columns (5) and (11) to (14) for working. The coefficients in *Table* I are multiplied by the values of K_1 or $\frac{1}{K_1}$ at the head of the column concerned before they are entered in *Table* II.

For each part of the table the values in columns (2), (3), and (4) are added horizontally and multiplied by the value in column (1); the product is entered in column (5), which is then totalled for each quantity. Similarly each line of columns (8), (9), and (10) is added, multiplied by the value in column (7), and the product entered in column (11). For each section column (11) is totalled, multiplied by the factor in column (6), and the product entered in column (12). Columns (5) and (12) are added to obtain the numerator in column (13), and column (14) gives the quantity concerned.

BENDING-MOMENT DIAGRAM AND SIGN CONVENTIONS.—The table shows the values of the bending moments to the left of the centre-line of the structure; anticlockwise moments are assumed to be positive, and vertical forces acting upwards and horizontal forces acting towards the left are also positive. As the structure and the loads are symmetrical about the centre-line, the bending moments to the right of the centre-line will be equal but of opposite sign to the bending moments at the corresponding joints on the left.

The bending-moment diagram can be plotted on the frame diagram as in

Fig. 1 in which the bending moment is shown on the tension side of each member. The bending moments at the joints are plotted from the table, the bending moments at the valley joints being zero, and the diagram completed by the parabolic bending-moment diagram due to the uniformly-distributed load on the rafters. As the horizontal projection of each rafter is $\frac{1}{2}L$, the maximum bending moment due to the distributed load will be $\frac{wL^2}{32}$ and not $\frac{wL^2}{8}$.

DEFLECTION.—As deflection may be of importance, the values of Δ_1 and Δ_2 (the deflections at B and E) should be calculated from the table. If they are greater than is permissible, stiffer rafters and columns are required. If the stiffness-ratios K_1, K_2, and K_3 are kept constant, the values in *Table* II will not change. The actual deflections are likely to be less than the calculated deflections.

CHOICE OF STIFFNESS-RATIOS.—The stiffness-ratios for which the bending moment in the columns will be lowest can be found by trial. If ϕ is assumed to be $\frac{3}{7}$:

If $K_1 = 2$, $K_2 = 1$, and $K_o = 1$: then $M = \frac{wL^2}{8}$, $-M_{AB} = 3{\cdot}07M$, $-M_{BA} = 0{\cdot}685M$, $-M_{TB} = 0{\cdot}96M$, $-M_{DC} = 2{\cdot}12M$, $M_{UC} = 1{\cdot}535M$, $-\Delta_1 = \frac{rs_o}{3EI_o} \times 3{\cdot}16M$, $-\Delta_2 = \frac{rs_o}{3EI_o} \times 4{\cdot}96M$ $\left(\text{since } h = \frac{7r}{3}\right)$.

If $K_1 = 1$, $K_2 = 1$, and $K_0 = 1$: then $M = \frac{wL^2}{8}$, $-M_{AB} = 2{\cdot}13M$, $-M_{BA} = 0{\cdot}93M$, $-M_{DC} = 2{\cdot}32M$, $M_{UC} = 1{\cdot}3M$.

If $K_1 = 0{\cdot}5$, $K_2 = 0{\cdot}5$, and $K_0 = 1$: then $M = \frac{wL^2}{8}$, $-M_{AB} = 1{\cdot}63M$, $-M_{BA} = 1{\cdot}04M$, $-M_{TB} = 0{\cdot}76M$, $-M_{DC} = 1{\cdot}38M$, $M_{UC} = 0{\cdot}7M$,

$$V_A = \frac{wL}{2} + \frac{1{\cdot}04M}{L}, \quad -\Delta_1 = \frac{rs_o}{3EI_o} \times 5{\cdot}18M, \quad -\Delta_2 = \frac{rs_o}{3EI_o} \times 6{\cdot}53M.$$

The condition that $K_1 = K_2 = 0{\cdot}5$ is usually the most suitable, so that for a given value of ϕ the table need be calculated only once.

Calculations.

The method can be applied to either steel or reinforced concrete frames. For example, the frame shown in *Fig.* 1 is required to support its own weight and an imposed load of 15 lb. per square foot of horizontal area (British Standard Code of Practice No. 3, Chapter V, Loading, 1952; non-access roof sloping at 30 deg.). The dimensions of the frame are $h = 14$ ft., $r = 6$ ft., $L = 20$ ft. 9 in., $s_o = 12$ ft., and the distance between frames $b = 16$ ft. $K_1 = K_2 = 0{\cdot}5$.

Steel Frame.

	per sq. ft.
Estimated weight of roof (including rafters)	= 13 lb.
Imposed load	= 15 lb.
Total	= 28 lb.

Load per foot of span w is $\dfrac{16 \times 28}{2240} = 0{\cdot}2$ ton per foot. If $K_1 = K_2 = 0{\cdot}5$, the greatest bending moment is M_{AB}, which has the value $1{\cdot}63M$.

$$M = \frac{wL^2}{8} = \frac{0{\cdot}2 \times 20{\cdot}75 \times 12}{8} = 128 \text{ in.-tons.}$$

Therefore $M_{AB} = 1{\cdot}63 \times 128 = 208$ in.-tons.

Also $V_A = \dfrac{wL}{2} + \dfrac{1{\cdot}04M}{L}$ + weight of stanchion and side frame

$$= \frac{0{\cdot}2 \times 20{\cdot}75}{2} + \frac{1{\cdot}04 \times 128}{12 \times 20{\cdot}75} + 0{\cdot}47 = 3{\cdot}08 \text{ tons.}$$

The fixing-moment at the foot of column AB increases the compressive stress in the extreme side. According to B.S. No. 449, the effective height of the column is $1{\cdot}5h = 252$ in., bending about the axis X–X.

For a stanchion 8 in. × 6 in. × 35 lb., $I_x = 115$ in.^{4}, $Z = 28{\cdot}6$ in.^{3}, $r_x = 3{\cdot}34$, $A = 10{\cdot}30$ sq. in. The actual direct stress is $\dfrac{3{\cdot}08}{10.30}$ (0·3 ton per square inch).

The permissible direct stress is 5·32 tons per square inch when $\dfrac{1{\cdot}5h}{r_x} = \dfrac{252}{3{\cdot}34} = 76$.

The stress due to bending is $\dfrac{208}{28{\cdot}6} = 7{\cdot}3$ tons per square inch.

As there is a point of inflection near the centre of the column the effect of lateral buckling can be neglected, and the permissible bending stress will be 10 tons per square inch. Since $\dfrac{0{\cdot}3}{5{\cdot}32} + \dfrac{7{\cdot}3}{10} = 0{\cdot}786$, which is less than one, 8-in. × 6-in. × 35-lb. stanchions are suitable. For the rafters, 10-in. × 6-in. × 40-lb. joists can be used for which $I = 204$ in.^{4}, giving

$$K_1 = \frac{I_1}{h} \times \frac{s_o}{I_o} = \frac{115}{14} \times \frac{12}{204} = 0{\cdot}49.$$

For $E = 30 \times 10^6$ lb. per square inch, $\Delta_1 = 0{\cdot}74$ in., and $\Delta_2 = 0{\cdot}88$ in. About half of these deflections will occur during erection, and the remainder only under exceptional conditions. The total deflection does not exceed 1/200 of the height of the column (change of slope equivalent to 1/400 of the span of a girder), which is satisfactory.

A frame comprising an 8-in. × 5-in. × 28-lb. stanchion and a 10-in. × 5-in. × 30-lb. rafter would be sufficient. The maximum deflection due to the dead load would be 0·65 in., and due to the total load the deflection would be 1·41 in. under abnormal conditions. These deflections will be reduced by the resistance at the valley joints and the stiffness of the roof and side covering, for which no allowance has been made.

Reinforced Concrete Frame.—Assume that the columns are 12 in. square and of 1 : 2 : 4 vibrated concrete, and the rafters 12 in. wide by 15 in. deep. Adding 250 lb. for the extra weight of the concrete frame to the vertical load calculated for the steel frame,

$$\text{Direct load} = 3{\cdot}08 \times 2240 + 250 = 7150 \text{ lb.}$$

$$-M_{AB} = 208 \times 2240 = 465{,}000 \text{ in.-lb.}$$

According to the tables for compression and bending of the Institution of Struc-

tural Engineers, for four $1\frac{1}{2}$-in. bars $d = 9{\cdot}75$ in., $p = 0{\cdot}06$, $e = \frac{465{,}000}{7150} = 65$ in. Therefore $\frac{e}{d} = 0{\cdot}67$ and $\frac{d_c}{D} = \frac{2{\cdot}25}{12} = \frac{1}{5}$ approximately. From the tables, when $\frac{d_c}{D} = \frac{1}{10}$: $n_1 = 0{\cdot}48$, $K_2 = 0{\cdot}45$; when $\frac{d_c}{D} = \frac{1}{8}$: $n_1 = 0{\cdot}485$, $K_2 = 0{\cdot}43$; when $\frac{d_c}{D} = \frac{1}{6}$: $n_1 = 0{\cdot}505$, $K_2 = 0{\cdot}39$, so that, by extrapolation, when $\frac{d_c}{D} = \frac{1}{5}$, $n_1 = 0{\cdot}51$, $K_2 = 0{\cdot}37$. For these values the stress in the concrete is 1070 lb. per square inch and in the steel 15,500 lb. per square inch. The allowable compressive stress for 1 : 2 : 4 (vibrated) concrete is 1100 lb. per square inch and the permissible tensile stress in mild steel is 18,000 lb. per square inch.

With 1 : 1 : 2 vibrated concrete and high-tensile steel the width might be reduced and a higher value of E used as, for high-grade dense concrete, E may have a value as high as 4×10^6 lb. per square inch. If $E = 3 \times 10^6$ lb. per square inch, $\Delta_2 = \frac{6{\cdot}53 r s_o M}{3EI_o} = 0{\cdot}64$ in., of which 0·3 in. is due to the dead load. This will be reduced by resisting moments due to friction and the stiffness of the walls and roof. If the low value of $E = 2 \times 10^6$ lb. per square inch is assumed, the deflection due to the full load on the roof is 1/330 of the height. Twice this deflection could usually be permitted, since it would be equivalent to a deflection of 1/325 of the span of a beam, which is the usual allowable deflection.

In a complete design allowance must be made for the wind pressure and suction on the walls and roof. This can be calculated from tables similar to *Table* I.

Derivation of the Formulæ for Table I.

The hinge C (*Fig.* 1) is prevented from moving horizontally by a horizontal force equal to the difference between the horizontal thrusts at C due to the loads on the frame CUE and on the frame ABTC. If this force is removed the joint C will move away from the central column EF by Δ_H, causing the thrust H_r in the frame CUE to be reduced by H_{ra}. The horizontal thrust H_{2C} in ABTC will be increased by an amount P, and there will be horizontal thrust H_C in column CD. There will be a similar effect at C since the frame is symmetrical, and the resultant horizontal thrust at E will be zero. Joint E will not move, and the load on EF will be wL.

From strain-energy equations it can be shown that

$$-H_1 h = \frac{wL^2}{8d_4}(12 + 30\phi + 8K_1\phi) = \frac{wL}{8d_2} \times f = hH_{2C} \quad . \quad . \quad (1)$$

in which H_{2C} is the thrust acting at C, and

$$-M_{AB} = \frac{wL^2}{8d_4}(4 + 15\phi + 8K_1\phi - K_1\phi^2), \text{ and } -M_{BA} = \frac{wL^2}{8d_4}(8 + 15\phi + K_1\phi^2),$$

in which the common denominator $d_4 = 8 + \frac{3}{K_1} + 18\phi + 24\phi^2 + 7K_1\phi^2$. The

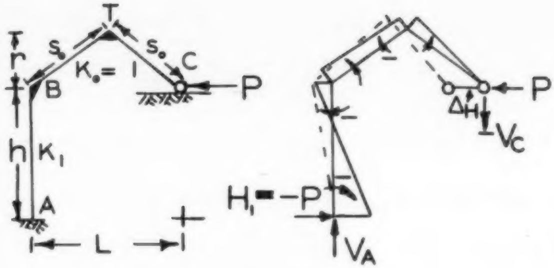


Fig. 2.

outward thrust of the frame CUE is $H_r = \dfrac{5wL^2}{8r}$ and is opposed by H_{2C}. The force required to prevent horizontal movement at C is $U = H_r - H_{2C}$. If C is allowed to move outwards by an amount Δ_H, the thrusts produced are

(*a*) A thrust in the column $H_C = \dfrac{3EI_2}{h^3}\Delta_H = \dfrac{3EI_o}{h^2 s_o}K_2\Delta_H$, since $K_2 = \dfrac{I_o}{h} \times \dfrac{s_o}{I_o}$.

(*b*) Reduction of thrust due to CUE, $H_{ra} = \dfrac{3EI_o}{2s_o r^2}\Delta_H$.

(*c*) Increase of thrust due to ABTC,

$$P = \frac{3EI_o}{h^2 s_o d_4}(12 + 8K_1)\Delta_H. \quad H_{2C} + P + H_C = H_r - H_{ra}.$$

Therefore $H_r - H_{2C} = U = P + H_C + H_{ra}$. Substituting in terms of Δ_H for H_C, H_{ra}, and P,

$$U = \frac{3EI_o\Delta_H d_5}{2h^2 s_o d_4 \phi^2} \text{ in which } d_5 = (24 + 16K_1)\phi^2 + d_4(1 + 2K_2\phi^2).$$

Also, from equation (1), $U = \dfrac{wL^2}{8}\left(\dfrac{5}{r} - \dfrac{f}{hd_4}\right)$ so that

$$\frac{3EI_o\Delta_H}{2hs_o} = \frac{wL^2}{8d_5}(5d_4 - f\phi).$$

The final thrust of the column $H_C = \dfrac{wL^2}{8hd_5}(10d_4 - 2f\phi)$ and $P = \dfrac{wL^2\phi}{8hd_4 d_5}(5d_4 - f\phi)$.

The final thrust in the frame ABTC, equal to the balancing thrust at A, is

$$H_1 = \frac{wL^2}{8hd_5}(12 + 150\phi + 88K_1\phi + 2K_2\phi^2 f).$$

The original thrust H_{2C} is increased by P due to the deflection Δ_H. The effect of P is to cause C to move to the left (*Fig.* 2) and therefore to increase the existing moments at A and B by

$$-M_{AB} = \frac{Ph}{6 + 4K_1}(3 + 4K_1 + 3K_1\phi) \text{ and } -M_{BA} = \frac{Ph(3 - 3K_1\phi)}{6 + 4K_1} = V_C L$$

in which V_C is the vertical reaction at C as shown. By substituting for P and adding the moments due to the load on frame ABTC when the deflection at C is prevented, the final values of M_{AB} and M_{BA} given in *Table* I are obtained.

Rigid and Flexible Joints.

If the actual moments at the joints are to correspond to the calculated values, care must be taken to ensure that the fixed joints are rigid and the hinged joints are flexible.

FIXITY AT BASE.—The columns are assumed to be rigidly fixed at the bottom. This can be effected, even when the bearing capacity of the ground is low, by placing the column and other loads on the base eccentrically, thus producing a moment counterbalancing the moment at the bottom of the column. It is not necessary to counterbalance this moment completely, but the centre of pressure should lie within the middle-third of the base, otherwise the base may tend to rotate.

Although it is usually ignored in designing a base, the ground at the sides of the base has a restraining effect, which may be substantial in stiff sub-soils and is appreciable even in filled-in ground, and will in many cases resist occasional loads of short duration, such as wind loads.

If the centre of pressure cannot be kept within the middle-third of a rectangular base, stiff tie-beams extending from A to A_1 in the plane of the frame can be used to resist the horizontal thrust, vertical loads, and the moments at the bottom of the columns.

JOINTS IN FRAMES.—The connections between the rafters at the ridges, and between the rafters and the external columns, must be sufficiently rigid to resist the bending moments at these joints. For reinforced concrete it is usually sufficient to increase the depth of the section, and consequently the lever arm. For steel frames, gussets should be provided and stiffened if necessary. For longer spans knee-braces may be required.

The joints between the rafters and the tops of the internal columns must be hinged, or sufficiently flexible to permit some rotation of the members at, or close to, the joints without the development of appreciable resisting moments. For precast concrete members steel seatings, welded to the main bars and bolted together centrally, can be provided, or bar-hinges or central dowels can be used. If the concrete is cast in place, jointed hinges with projecting bars can be set in the concrete, or the moment of resistance can be reduced by placing the reinforcement at the centre-line at the hinge. In steel frames pin-joints can be used, but ordinary bolted or riveted joints are preferable as the small restraint which they exert does not appreciably affect the distribution of moments in the frame but does reduce the horizontal deflection. These joints should have sufficient rivets to resist the vertical and horizontal shearing forces, placed as near to the centre-line of the member as possible. Cleats, plates, and splices should be in the web and not the flange.

Application of the Method to Other Frames.

Although no particular economy is claimed for this type of frame, a solution is given that requires very little calculation, and, due to constructional rigidities, etc., the design is usually well on the safe side. For much more economical design, similar tables can be prepared for frames with stiff-jointed fixed bases and with stiff-jointed hinged bases which require very little more calculation. The maximum bending moments and deflections derived are, however, much

smaller and the resulting design should show a similar saving in materials and cost. This is demonstrated in the following.

Comparison of frames acted upon by identical uniformly-distributed loading for the ratios: $K_0 = 1$, $K_1 = 0{\cdot}5$, $K_2 = 0{\cdot}5$, $M = \frac{wl^2}{24}$. All values are coefficients of M. (1) Rigid frame with valley hinges, (2) Rigid frame, (3) Rigid frame with hinged bases.

	M_{AB}	M_{BA}	M_{DC}	M_{CD}	M_{TB}	M_{EU}	M_{UC}	M_{CU}	M_{CT}	Δ_1	Δ_2
(1)	4·89	3·120	4·140	0·000	2·280	0·00	2·220	0·00	0·00	$5{\cdot}18\frac{rs_0}{EI_0}$	$6{\cdot}530\frac{rs_0}{EI_0}$
(2)	1·09	1·245	0·235	0·138	0·630	1·32	0·368	1·61	1·47	0·7 ,,	0·250 ,,
(3)	0·00	1·160	0·000	0·688	0·975	1·63	0·680	1·945	1·89	1·0 ,,	0·505 ,,

Maximum bending moments:

case (1)—$M_{AB} = 4{\cdot}89$. $M_{BA} = 4{\cdot}14$. $\Delta_2 = 6{\cdot}53\, rs_0 \div EI$.
case (2) Between C and U, 1·738. $\Delta_1 = 0{\cdot}70$,,
case (3) Between C and T, 2·285. $\Delta_1 = 1{\cdot}00$,,
Between C and U, 2·07.

The rigid frame is therefore the stiffest and most economical.

Tables similar to *Table* I can be prepared for all types of gable or rectangular frames, with one or more bays and with fixed, hinged, or semi-rigid connections at the supports. For any particular type of frame, including single-bay and two-bay frames with fixed joints, tables can be produced showing the effect of horizontal and vertical loads, applied moments (such as are caused by crane girders), temperature variation, and settlement of the supports.

Effect of Shape of Test Specimens on Compressive Strength.

In Bulletin No. 122 of the Deutscher Ausschuss für Stahlbeton (Berlin: Wilhelm Ernst & Sohn. Price 14 D.M.) Professor Kurt Walz describes tests to determine the effect of the shape of concrete specimens upon the compressive strength compared with 8-in. cubes. Eight different shapes were tested, each having a cross-sectional area of 900 sq. cm. and a depth of 40 cm. The shape of the specimens and the average of the results of the tests are given in the table.

ALL DIMENSIONS IN CENTIMETRES		STANDARD CUBE 20CM X 20CM	RECTANGLES 50 X 18	RECTANGLES 72 X 12·5	RECTANGLES 100 X 9	CIRCULAR ARC 100 X 9	HOLLOW BOX SECTION 50X50X5	HOLLOW CIRCLE 50 DIA X 5	TEE SECTION 72 X 9 28 X 9	CHANNEL SECTION 72 X 9 14 X 9
COMPRESSIVE STRENGTH AFTER 16 TO 24 DAYS	KG. PER SQ. CM.	451	399	394	365	395	398	410	387	386
	LB. PER SQ. IN.	6440	5670	5600	5190	5630	5670	5840	5500	5490
PROPORTION OF CUBE STRENGTH		1·00	0·88	0·87	0·81 (MIN)	0·88	0·88	0·91 (MAX)	0·86	0·86

An Unusual Water-Tower in France.

BUILT to supply a housing estate on the outskirts of Caen, in Normandy, the water-tower (see *Figs.* 1 and 2) has several unusual features. The tank, of 650,000 gall. capacity, has arched walls spanning between abutments formed by raking columns, a base consisting of sixteen curved slabs spanning on to radial beams, and a shell roof comprising sixteen conoidal elements. The sixteen raking columns and a central circular column meet at the base of the tower on a single foundation. To provide lateral stability to the structure, beams extend from the columns, near the bottom of the tower, to L-shaped frames set out on an ellipse around the tower. These frames also support single-story offices accommodating administrative and technical services for the estate. Spanning between the stabilising beams are conoidal shells forming the roof of a covered market.

A cross section (*Fig.* 1) shows the arrangement of the beams and columns. There are hinges at the central support, at the junctions of the stabilising beams and L-frames, and at the tops of the raking columns where they join the radial beams at the base of the tank. The hinge at the central support is spherical and of the Freyssinet type ; it supports a load of 5000 tons at a working compressive stress in the concrete of 4750 lb. per square inch. The base is 25 ft. in diameter and is carried down to limestone about 20 ft. below ground level. Prestressed ties are used around the top and bottom of the tank ; in addition there are two ties near the bottoms of the raking columns.

The wind pressures for which the tower is designed vary from 18 lb. per square foot at a height of 80 ft. to 14 lb. per square foot at a height of 30 ft. The maximum compressive stress in the concrete is 1000 lb. per square inch, except for local stresses at the hinges, and the

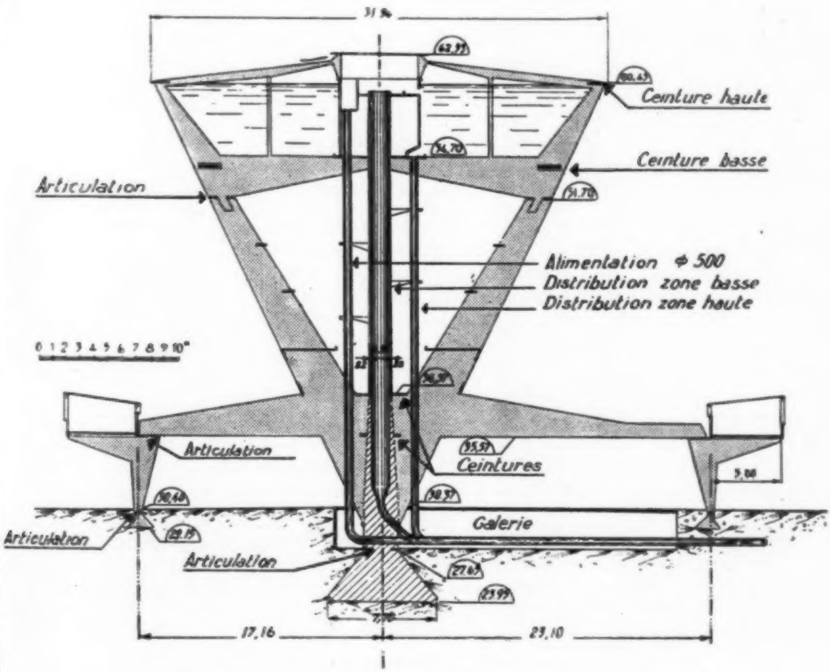


Fig. 1.—Cross Section showing Main Members.

Fig. 2.—Combined Water-tower, Offices, and Market.

tensile stress in the steel is 30,000 lb. per square inch. The volume of concrete used is 41,000 cu. ft. and the weight of steel reinforcement 150 tons. The walls of the tank are 4 in. thick and were concreted with a cement gun.

The architect is M. Gillet and the engineer is M. Sarger. The structure was built by Entreprise S.E.T.A. The foregoing is abstracted from an article by M. A. Darde in " Travaux " for February, 1958.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering," September–October, 1908.*

The London Building Act.—We are glad to see that there will at last be some movement on the part of the London County Council to obtain a consolidation and general amendment of the existing Building Acts which in many respects are antiquated. . . . Reinforced concrete was for all practical purposes non-existent in Great Britain in 1894, when the Building Act was last thoroughly overhauled, and it is high time, now that this form of construction has become popular and is being used with great advantage by our Government departments and other great public bodies, that the London building owner should also be able to make full use of the material.

Concrete in the U.S.A.—At the recent conference of governors and scientists at the White House, Washington, which was held under the chairmanship of President [Theodore] Roosevelt, there was a discussion on the conservation of the natural resources of the United States, in the course of which Mr. Andrew Carnegie, speaking of iron, said: " The next great use of iron is in construction, especially of buildings and bridges. Fortunately the use of concrete, simple and reinforced, is already reducing the consumption of structural steel. The materials for cement and concrete abound in every part of the country, and, while the arts of making and using them are still in their infancy, the products promise to become superior to steel and stone in strength, durability, convenience, economy and use."

* "Concrete and Constructional Engineering ' appeared in alternate months until September, 1909.

Precast Concrete in an Office Building.

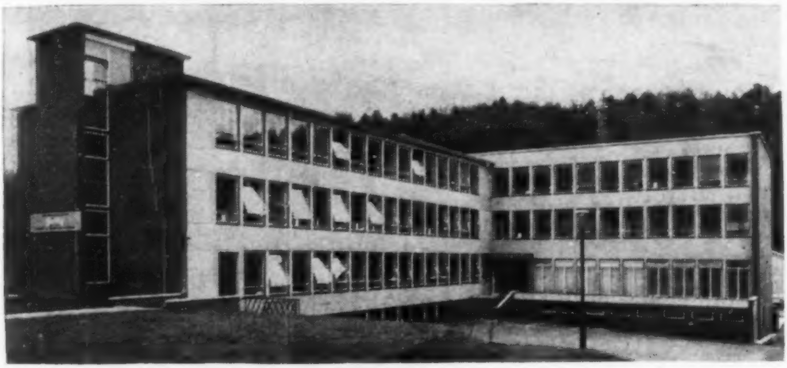


Fig. 1.

A NEW office building (*Fig.* 1) at Whyteleafe, Surrey, for the National Canning Co., Ltd., is L-shaped in plan and has three stories, each 10 ft. 3 in. high, and a semi-basement. The wings are 96 ft. and 107 ft. long and 35 ft. wide.

Reinforced concrete columns support the floor slabs and a shallow reinforced concrete beam. The soffit of the slabs is smooth, except for the central beam which projects 4 in.; the beams at the perimeter are within the depth of the floor. The floors and roof consist of prestressed precast planks and hollow clay tiles, with a topping cast in place. The overhanging eaves were precast and incorporated in the roof.

The external columns are 12 in. by 6 in. in cross section and at 5 ft. 3 in. centres. The internal columns supporting the central beam are at 10 ft. 6 in. centres and form two bays 18 ft. 4½ in. and 15 ft. 4½ in. wide. The internal and corner columns were cast in place.

The remainder of the external columns were precast in pairs, together with part of the spandrel wall, to form panels one story in height (*Fig.* 2). The wall portion of each panel extends on both sides of the columns for a distance of half a panel. They consist of 3 in. of reinforced concrete lined with wood-wool slabs 2 in. thick which were plastered after erection. The wood-wool slabs were placed in the bottom of the mould when the panels were cast.

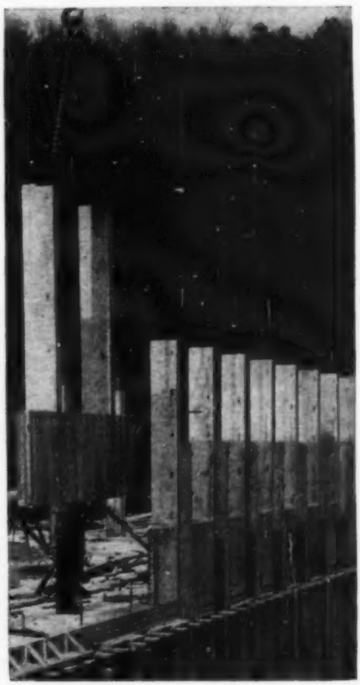


Fig. 2.

Fig. 3.

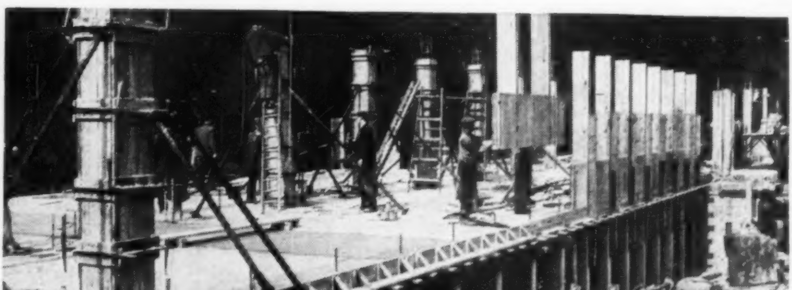


Fig. 4.

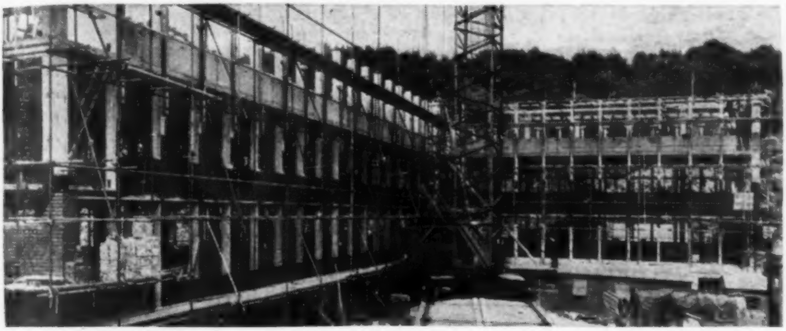


Fig. 5.

Dovetailed slots were cast in the wall part of the panels and the columns to allow for the fixing of the precast facing slabs, which are 2 in. thick and have an exposed-aggregate surface of white Derbyshire spar and grey Cornish granite (*Fig.* 3).

The panels were precast in a factory, delivered by road, and placed in position directly from the lorry by means of a tower crane. They are connected to the floors by dowelled and grouted joints; dowels were cast into the tops of the columns, and pockets with grouting ducts were formed in the bottoms of the columns. For handling purposes, the top few inches of each dowel were threaded to receive a nut which secured a piece of steel channel to which crane slings were attached.

To allow the panels to be erected on the upper stories before those in the story below had been grouted, holes were cast in the columns so that short timber spacers could be bolted between the two stories; it was found, however, that the unloading and erection of the panels for each story could be carried on continuously. Joints in the spandrel walls mid-way between alternate pairs of columns were formed as grouted recesses; strips of galvanised mild steel were inserted to strengthen the joints and act as water-stops.

Shuttering was required only for the 12-in. by 4-in. projections of the central beam, the columns cast in place, and the edges of the floor slabs. The erection of the frame was completed in eleven weeks; *Figs.* 4 and 5 show the progress made in four weeks.

The architects were Messrs. Stock, Page, & Stock, and the general contractors Messrs. Rush & Tompkins, Ltd. The frame was designed and constructed by the London Ferro-Concrete Co., Ltd., and the facing slabs were made by Messrs. John Ellis & Sons, Ltd.

The Properties of Twisted Square Bars.

In a brochure of 36 pages the Square Grip Reinforcement Co. (London), Ltd., has reprinted the following papers read before the Institution of Structural Engineers: "Tests on Concrete Columns Reinforced with Square Twisted Steel and Mild Steel", by Dr. K. Hajnal-Kónyi; "Ultimate Strength of Axially-loaded Columns Reinforced with Square Twisted Steel and Mild Steel", by Professor R. H. Evans and K. T. Lawson; and "Tests on Eccentrically-loaded Columns with Square Twisted Steel Reinforcement", by Professor R. H. Evans and K. T. Lawson. Copies of the brochure may be had (free) from the office of the Company, Colnbrook By-pass Road, Colnbrook, Slough, Bucks.

Lightweight Reinforced Concrete.

The properties of reinforced concrete made with expanded-shale aggregates, and the constants and formulæ required for the design of lightweight reinforced concrete, are given in two booklets issued by the Butterley Co. Ltd., of Ripley, Derbyshire, from whom copies can be obtained free of charge.

TENDERS INVITED.

CORK COUNTY COUNCIL
IRELAND

CONSTRUCTION OF PRE-STRESSED CONCRETE BRIDGE near YOUGHAL TOWN

The Cork County Council invites tenders from Civil Engineering Contractors experienced in bridge construction and in underwater works, for the building of a New Prestressed Concrete Bridge over the River Blackwater near Youghal Town.

The Bridge is approximately 33 feet wide and 1242 feet long and consists of:

(*a*) 5 Spans of 880 feet overall length with span arrangement 146′—196′—196′—196′—146′. The central span of 196′ is divided into two cantilever spans of 58 feet and a suspended span of 80 feet.

(*b*) Embanked approaches of about 132 feet at the Western side and 230 feet at the Eastern side including about 50 feet of suspended roadway adjoining each shore span.

The four river piers are carried on 5 feet diameter concrete-filled cylinders to rock. The suspended roadway adjoining the shore spans is carried by wing walls directly on rock.

In addition to tendering on the scheme prepared by the Council's Consulting Engineer, Mr. W. J. L. O'Connell, M.E., M.Inst.C.E.I., F.R.I.C.S., Contractors who so desire may also submit alternative foundation proposals. Contractors are also permitted on their own responsibility to tender for a prestressed concrete bridge on their own design.

The Council does not bind itself to accept the lowest or any tender or any tender based on any alternative scheme or design. The time proposed for completion will be taken into account in deciding on the award of the Contract. The acceptance of any tender will be subject to the approval of the Minister for Local Government.

Application for copies of the documents, "Specification", "Conditions of Contract", "Bills of Quantities", "Drawings" and "Conditions for Alternative Proposals" should be made to the Council's Consulting Engineer, Mr. W. J. L. O'Connell, M.E., M.Inst.C.E.I., F.R.I.C.S., 9 South Mall, Cork, accompanied by a deposit of £50. 0. 0. (returnable after receipt of a bona fide tender not subsequently withdrawn).

The documents may be inspected at the Office of the Consulting Engineer, Cork.

Tenders on the prescribed form (unaltered in purport) signed and sealed and endorsed with the Contractor's Name and the words: "Tender for Youghal Bridge" should be lodged with the Secretary, Cork County Council, Courthouse, Cork, Ireland, not later than *12 o'clock noon on Saturday 17th January, 1959*. Separately sealed Bills of Quantities, fully priced and extended and totalled in ink and endorsed with the name of the Contractor and the words: "Priced Bill of Quantities for Youghal Bridge" should be lodged at the same time, otherwise the tender will not be considered bona fide. The sealed packages containing the priced Bills of Quantities will be returned unopened to the unsuccessful Contractor on application. The Contractor whose tender is accepted will be required to enter into a formal contract with the Cork County Council and to give a satisfactory Bond for the performance of the Contract as provided for in the Conditions of Contract.

Prospective Contractors are to furnish evidence of their experience and competence in this class of work.

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